GOLDEN GATE BRIDGE AND HIGHWAY DISTRICT

SAN FRANCISCO, CALIFORNIA

REPORT ON PROPOSED INSTALLATION OF RAPID TRANSIT TRAINS ON GOLDEN GATE BRIDGE

By

Engineering Board of Review

OTHMAR H. AMMANN

FRANK M. MASTERS

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April 10, 1962

To the Honorable Board of Directors
Golden Gate Bridge & Highway District
Box 9000 Presidio Station
San Francisco, California

Gentlemen:

We have the honor to submit herewith our report on a review which you authorized us to undertake in order to resolve the question of whether or not installation of rapid transit on the Golden Gate Bridge is practicable or advisable.

Our conclusion is that rapid transit installation on the bridge would encroach on the safety of the structure and lead to a decrease in its life. We therefore report that said installation is not advisable.

We trust that the basis for our conclusions and recommendations contained in the report will permit you to arrive at a definite decision on the question of rapid transit traffic. You may wish to abandon the more detailed study and investigation, originally contemplated in a second phase of our study.

We take this opportunity to express our appreciation of having been asked to render this service and of the very helpful assistance we have received on the part of your General Manager, Mr. James Adam, and his staff.

Respectfully submitted,

O. H. AMMANN

FRANK M. MASTERS

N. M. NEWMARK

REPORT BY ENGINEERING BOARD OF REVIEW ON FEASIBILITY OF RAPID TRANSIT INSTALLATION ON GOLDEN GATE BRIDGE

INTRODUCTION AND SCOPE OF REVIEW

On January 12, 1962 the Board of Directors of the Golden Gate Bridge and Highway District appointed the undersigned as an Engineering Board of Review with instructions to review conflicting reports which had been made on the feasibility and advisability of installing rapid transit over the Golden Gate Bridge as a part of the proposed rapid transit system in the San Francisco Bay area.

In a report of March 1961 to the San Franciso Bay Area Rapid Transit District Mr. C. H. Gronquist, Consulting Engineer, found the addition of two rapid transit tracks on the bridge feasible and practicable.

At the request of the Golden Gate Bridge and Highway District Mr. Clifford E. Paine, Consulting Engineer, reviewed Mr. Gronquist's report and, in a report of July 1961, recommended that rapid transit trains not be permitted to operate over the bridge.

On February 5, 1962 we submitted to the Board of Directors of the Golden Gate Bridge and Highway District a proposed scope of our investigations. This scope, approved by the Board of Directors, provided, as a first phase, substantially for the following studies:

Determination of the load conditions imposed upon the bridge by rapid transit trains and the necessary additions to and alterations of the bridge structure.

Determination of the resulting stresses in the major carrying parts of the bridge, viz. cables, anchorages, towers, suspenders and stiffening trusses.

Review of fatigue effects on the structure caused by additional stresses and stress fluctuations, and the conduct of fatigue tests on cable wire.

In undertaking this investigation we understood that it was the prime concern of the Board of Directors of the Golden Gate Bridge and Highway District that, whatever changes and additions are contemplated, the bridge must be preserved in a first class condition, with a conservative margin of safety, to serve highway traffic for which it was intended and designed.

It is in this sense that our investigation has been conducted and our findings and conclusions are reported herewith.

Comments on possible fatigue effects in Golden Gate Bridge by N. M. Newmark and W. H. Munse are contained in an appendix to this report.

SUMMARY OF FINDINGS AND CONCLUSIONS

As originally designed and built the Golden Gate Bridge had a certain reserve capacity to carry additional load. It resulted from a very liberal assumption of intensity of moving load on the roadway and sidewalks and from the conservative stresses permitted in its major carrying members. All structural features were designed in accordance with best practice and the bridge has since been maintained in good condition.

It is therefore possible to place limited additional loads on the existing structure and still preserve a conservative stress condition. Part of the original margin of carrying capacity has been utilized by structural features, such as the lower lateral wind bracing and tracks for the traveling maintenance platforms which were added later.

Furthermore we consider it advisable to reserve a moderate margin for possible future structural changes and additions for highway purposes such as have become necessary or desirable in course of time on most large bridges.

Installation of rapid transit would add more dead weight to the structure, including the tracks, structural members to support the tracks and reinforcement of the existing deck structure and stiffening trusses. We have estimated this superimposed dead weight, but must make the reservation that more detailed studies might disclose the necessity for further increases.

To offset this dead weight required by rapid transit installation, at least in part, we have assumed as a permissible modification of the present structure the reduction of the width of the sidewalks from 10 to 6 feet. We do not recommend, however, the replacement of the present solid deck slabs by open steel grating in any of the roadway lanes or the sidewalks.

For the moving load on the tracks we have assumed light-weight rapid transit trains as contemplated by the San Francisco Bay Area Rapid Transit District. We made this assumption, however, with the reservation that such trains may eventually prove impracticable and heavier ones might have to be used.

Light-weight stainless steel cars, having a loaded weight about two-thirds of that of the standard subway cars in use heretofore, have been operated on a

branch of the Philadelphia rapid transit system since early 1961. However, there appears to be a controversy as to whether such light cars will prove satisfactory over longer periods.

If heavier cars should become necessary, they would increase not only the moving load, but also the weight of the structure which has to carry them.

On the basis of the aforementioned dead weight and moving load assumptions, we have calculated the resulting stresses in the major carrying members, the main cables, anchorage chains, tower shafts, suspenders and stiffening truss chords.

It is our judgment that, in case rapid transit is installed, the stresses in the wire cables can be conservatively permitted to go about 5% higher than the limiting value set up in the original design specifications, but that those set up for the other members, anchorage chains, towers and stiffening trusses, should not be exceeded. Higher stresses would impair the life or adequacy of the structure.

In setting up these limits we have given consideration to a number of factors which tax the structure more severely under rapid transit operation than under exclusively highway traffic. Such factors include greater deformations of the suspended structure and the towers under traffic, with resulting greater and more frequent secondary stresses at connections of members. These greater and more frequent stress fluctuations, together with greater impact effects from trains, influence the fatigue of the materials as well as of the connections.

The stress figures given in the text of this report disclose that, even under light-weight trains and the reasonably conservative load conditions we have assumed, the permissible stresses would be appreciably exceeded in all major carrying members. The margin of safety would be decreased to a greater extent than is advisable in such an important structure designed to last for a long time.

From available information on fatigue effects resulting from repeated stresses or stress fluctuations we conclude that installation of rapid transit on the Golden Gate Bridge would lead to a decrease in the possible life of the bridge.

There are other considerations which may render the advisability or practicability of rapid transit installation on this bridge questionable. As described more in detail in the text of this report they concern the difficulties and ramifications involved in the necessary adjustments, additions to, and modifications of the deck structure, stiffening trusses, suspenders, expansion dams and pylons of the suspension spans, while the bridge is under traffic.

The alterations of the arch span on the San Francisco side, the Marin anchorage and the curved approach viaducts would also involve difficult, if not impracticable, structural changes and reinforcements.

All these very complex and delicate operations would require far more detailed studies than have been made so far before their practicability can be ascertained.

Further studies might also disclose that, due to the flexibility of the structure and its susceptibility to rapid lateral, vertical and torsional deformations under the passage of high-speed trains, in particular the angularity of the track alignment and profile at the towers, the bridge is unsuited for rapid transit operation.

We have made no attempt to estimate the cost involved in installing rapid transit on the bridge, but it appears to us that the figures heretofore reported would be greatly exceeded.

In view of the results of our studies we have to conclude that installation of rapid transit on the Golden Gate Bridge is not advisable.

O. H. AMMANN

FRANK M. MASTERS

N. M. NEWMARK

In support of the aforementioned conclusion we submit the following detailed investigations and findings.

LOADS AND FORCES ON SUSPENSION SPANS

Dead Load

As originally built the suspension spans were designed for an assumed uniform dead load of the structure suspended from the towers of 21,000 lbs. per lin. ft. of bridge (Chief Engineer's Report of September 30, 1937 p. 81).

As actually completed in 1937 the dead load was estimated to be 21,300 lbs. in the center span and 21,500 lbs. in the side spans (same reference).

In subsequent years there were added a second or bottom lateral system weighing 1,400 lbs. per ft. of bridge and supports for maintenance platforms weighing 200 lbs. per ft. (C. E. Paine Report of July, 1961). These additions increased the dead load of the center span to its present value of 22,900 lbs. per ft. of bridge.

In most large bridges changes and additions have been made in the course of time, partly for desirable or necessary improvements, partly to accommodate additional facilities and constantly increasing vehicular loads within the available safe margin. In the case of the Golden Gate Bridge such additions could embrace possible additional stiffening such as stays against aerodynamic action (Report of Board of Engineers on Alterations of Golden Gate Bridge of January 1953), gantry frames to support traffic control lights, water supply lines (Report by Sverdrup, Parcel & Associates, Inc., of October 1961), utility ducts, repainting of steelwork and possibly resurfacing of roadway slabs on account of wear or disintegration (such resurfacing of concrete pavements has been done on or is being considered for other bridges). It would be advisable to maintain a reserve for such possible future additional dead loads to the extent of at least 800 lbs. per ft. of bridge before additions of rapid transit facilities are made.

The total dead load of the center span with this modest reserve would amount to 23,700 lbs. per ft. of bridge. Because of the very liberal live load and the conservative permissible unit stresses assumed in the original design, this increased dead load would still leave a reasonably conservative margin of safety, even without compensating reductions in weight in the present structure.

In considering the addition of rapid transit it is evident that partially compensating reductions in the weight of the existing structure would have to be made. Such a reduction could be effected, without appreciably imparing the usefulness of the highway facilities, by reducing the width of the pedestrian walks from the present 10 ft. to say 6 ft.

It would not be advisable, however, to substitute open gratings for the present solid flooring in either the sidewalks or any of the roadway lanes. Such open grating decks are decidedly disadvantageous from the point of view of vehicle passengers and pedestrians. They would depreciate the character of the bridge as a high class highway facility.

The reduction in the width of the sidewalks with corresponding setting back of the railings and other changes would effect a weight reduction of approximately 1,000 lbs. per ft. of bridge.

If two rapid transit tracks are installed in the center in the plane of the bottom lateral system, the following estimated additions would have to be made in the dead load of the suspended spans per lin. ft. of bridge:

For track structure (rails, ties, etc.), railings and skid girders (as protection against possible derailment of trains) 1,365 lbs.

For new structural members to support the tracks (stringers, bracing between stringers, etc.) 535 lbs.

For changes and additions to existing members of the deck structure (cross-frames, lateral bracing, suspender connections, stiffening truss reinforcements, etc.) 1,100 lbs.

These additions and changes in the dead load due to installation of two rapid transit tracks are based on the weight of the rolling equipment hereinafter described under "Rapid Transit Live Load." They also allow for necessary structural features in the deck structure to allow adequately for lateral, braking and torsional forces produced by the rapid transit trains.

There may be additional dead load resulting from the following most necessary reconstruction. The central portion of the reinforced transverse struts, as proposed by Mr. Gronquist, is in our opinion very objectionable. It forms a rectangular tube only 14 inches wide and 3 feet high, with a few small handholes provided in the sidewebs. This would not permit adequate inspection and maintenance on the inside. Since, with the addition of rapid transit, these struts become major carrying members, supporting the track stringers and forming the lower chords of the rigid crossframes, it is particularly important that they can be properly maintained against corrosion.

A proper design of these struts might also affect the design of the laterals and the connections to the lower chords of the stiffening trusses and the diagonals of the wind truss. Such redesign of the entire lower deck would further increase the dead load appreciably.

These changes, additions and the above mentioned moderate reserve dead load, exclusive of the possible additional weight mentioned previously bring the

total design dead load in the center span to 25,700 lbs. per ft. of bridge, which is 4,700 lbs. per ft. or 22.4% higher than the original design dead load of 21,000 lbs. per ft. of bridge.

Design Live Loads

The adoption of appropriate design live loads on a long-span bridge, carrying multiple lanes of traffic and comprising a large number and variety of load units, is largely a matter of established practice and individual judgment. However, viewed in proper perspective, moderate differences in this respect are inconsequential in their effect on the cables, anchorages and towers, when compared to the effect of the combined dead and live loads and other forces.

Highway Live Load for Cables, Anchorages and Towers

The maximum direct or axial stresses in the cables, anchorages and towers are governed by a uniform live load of unrestricted length covering the center and side spans. The maximum combined axial and bending stresses in a tower are produced by a uniform load covering the center span and the far side span.

For the original design of the bridge for highway traffic the following uniform live load per ft. of bridge was assumed.

On the six lane roadway	3,000 lbs.
On two ten ft. sidewalks	1,000 lbs.
Total original highway design live load . . .	4,000 lbs.

Compared to design live loads used on other long-span bridges this load may be regarded today to be very conservative, especially the load on the sidewalks. However, it established a desirable reserve for possible future changes or additions.

In his report of March 1961 Mr. Gronquist assumed a roadway live load of 2,600 lbs. per ft. of bridge for loaded lengths exceeding 1,200 ft. and neglected load on the sidewalks. This must be considered too low, especially in view of the less conservative unit stresses he considers permissible in case rapid transit is added. For loaded lengths less than 1,200 ft. he assumed 3,260 lbs. per ft. of bridge.

Mr. Paine accepted the load of 2,600 lbs. per ft. on the roadway for lengths in excess of 1,200 ft., but properly added 300 lbs. per ft. for the two 10 ft. sidewalks, making the total highway live load of unrestricted length 2,900 lbs. per ft. of bridge. For load lengths shorter than 1,200 ft. he assumed 3,300 lbs. per ft. of bridge.

We consider a live load of unrestricted length of 2,700 lbs. per ft. (450

lbs. per ft. per lane) on the six-lane roadway, plus 200 lbs. per ft. on two sidewalks of 6 feet width, or a total of 2,900 lbs. per ft. of bridge appropriate and reasonably conservative when combined with the rapid transit live load hereinafter recommended. This is 1,100 lbs. per ft. less than the original design load of 4,000 lbs. per ft. and more nearly equivalent to the highway loads used for the design of other long-span bridges. Impact and reductions for multiple lane loads are included in all highway live loads.

Rapid Transit Live Load for Cables, Anchorages and Towers

The original design live load of the Golden Gate Bridge did not make allowance for rapid transit.

Mr. Gronquist based the rapid transit design live load on the assumption that high-speed, light-weight, trains will be operated over the Golden Gate Bridge. He assumed a typical train to be composed of ten cars, each 67′-3″ long, weighing empty 65,460 lbs. and 88,260 lbs. with passengers. This represents a uniform load of 1,310 lbs. per ft. per track. To this he added 7% for impact, making the total rapid transit design live load 1,400 lbs. per ft. of track or 2,800 lbs. per ft. of bridge.

He limits the length of this load, however, to that of two trains or to 1,360 ft. This is proper for determining the stresses in the stiffening trusses, but if placed in the most unfavorable position to cause maximum stresses in the cables, anchorages and towers, it is approximately equivalent to a uniform load of unrestricted length of only 1,300 lbs. per ft. of bridge.

We agree with Mr. Paine that under possible actual operating conditions this restriction is not realistic. Based on the same light-weight train, but of extended length, Mr. Paine assumes a rapid transit design load of 1,100 lbs. per ft. per track or 2,200 lbs. per ft. of bridge.

An effective control system could undoubtedly restrict the rapid transit train loading, but it would not be advisable to rely on it at all times. In the course of time when traffic becomes denser or in cases of emergency such control might be modified.

Assuming that such light-weight trains may eventually prove satisfactory under operation we would consider a design load of 1,700 lbs. per ft. of bridge of unrestricted length and, inclusive of impact, adequately conservative and have assumed this load for this investigation. In making this assumption, however, we must call attention to the fact that such light trains have not yet stood the test of time under actual operation.

Light-weight stainless steel cars weighing approximately 1,400 lbs. per ft. when loaded to capacity, have been in operation by the Philadelphia Transpor-

tation Co. since early 1961. So far they have reportedly given satisfactory service. However, this is too short a time to permit definite conclusions as to their adequacy under long-time operation and they are therefore still in an experimental and apparently controversial stage. We understand that, after many years of experimenting with lighter than present standard rolling equipment (which weighs between 2,100 and 2,200 lbs. per ft.) the New York Transit Authority is about to put into experimental operation a few cars weighing fully loaded 1,810 lbs. per ft. per track. With a 7% impact effect this would be 1,930 lbs. per ft. per track, or 38 percent more than the very light trains we have assumed.

In assuming the light-weight rapid transit trains for the purpose of this investigation, we therefore do so with the reservation that this load basis may eventually prove inadequate and must be re-examined before commitments to install rapid transit on the Golden Gate Bridge are made. Furthermore, a careful investigation as to whether these very light cars could not be overturned by severe local gusts of wind when they are standing on or passing over the bridge would be advisable. In any case we consider it essential that adequate protective measures against such happenings, as well as against possible derailments, be taken.

Combined Design Live Load for Cables, Anchorages and Towers

The above highway and rapid transit live loads assumed for this study give a combined live load of 2,900 + 1,700 = 4,600 lbs. per ft. This total design live load for combined highway and rapid transit is 15% in excess of the original design load of 4,000 lbs. per ft. of bridge.

For comparison a corresponding combined live load of unrestricted length of 2,600 + 1,300 = 3,900 lbs. per ft. of bridge was used by Mr. Gronquist and a combined load of 2,900 + 2,200 = 5,100 lbs. for load lengths in excess of 1,200 ft. was used by Mr. Paine.

Design Live Loads for Stiffening Trusses

The maximum stresses in the stiffening trusses are caused by a live load of restricted length. In accordance with established practice, and as applied to the design of other long-span bridges with multiple lanes of traffic, a greater load intensity is properly used for loads of restricted length.

For the purpose of this study the following live load assumption is made: a highway live load of 3,200 lbs. per ft. of bridge of a length of 1,200 ft. or less, plus a rapid transit live load of 2,800 lbs. per ft. of bridge, also of a length of 1,200 ft. or less. This restricted length live load of 3,200 + 2,800 = 6,000 lbs. per ft. of bridge is placed in a position to cause maximum stress in the stiffening trusses.

This compares with 3,260 + 2,800 = 6,060 lbs. per ft. of bridge on a restricted length assumed by Mr. Gronquist and with 3,300 + 2,800 = 6,100 lbs. on a restricted length assumed by Mr. Paine.

Design Live Loads for the Deck Structure and Suspenders

For the design of the stringers carrying the rapid transit tracks and for the examination of the other members of the deck structure and the suspenders the previously mentioned light-weight rapid transit cars, weighing with passengers 88,260 lbs., or 22,065 lbs. per axle, have been assumed, with impact and permissible unit stresses for the deck structure in accordance with the specifications of the American Railway Engineering Association of 1961. The permissible stresses in the suspenders are covered hereinafter under "Suspenders."

Temperature Changes and Wind Forces

The assumptions made in this study to determine the effects of temperature changes and static wind forces are substantially the same as those made by Messrs. Gronquist and Paine and conform to generally accepted practice for long-span bridges.

For the cables, anchorages, towers and stiffening trusses a temperature change of 50°F. was used. The assumed static wind pressure is 30 lbs. per sq. ft. of area of exposed deck structure, plus 300 lbs. per lin. ft. of bridge on trains, plus 50 lbs. per sq. ft. of exposed area on the towers.

For the various combinations of temperature changes and wind forces with dead and live loads higher permissible unit stresses are assumed than for dead load and full live load combined. This will be covered more fully under "Stresses."

Aerodynamic Action

We have studied the reports of November 30, 1960 by Mr. George S. Vincent and Mr. Ernest G. Wiles and the report of February 23, 1961 by Prof. F. B. Farquharson on the wind tunnel tests made on section models to determine the effect of rapid transit trains and of changes in the deck structure on the behavior of the Golden Gate Bridge under aerodynamic action.

In interpreting the results of these tests it must be kept in mind that there is a great difference between action by a uniform, steady and prolonged wind stream on a small section model in a wind tunnel and actual wind action on a long-span heavy suspension bridge structure, especially under higher wind velocities. Refinements in the form of a structure, such as shape of railings, curbs, etc., even the addition of rapid transit on the bridge, have relatively little effect on the behavior of the actual structure.

This is indicated by the fact that the section models do not indicate any appreciably different critical wind velocities under such structural variations. (Report of F. B. Farquharson, Table 3a and Report of Messrs. Vincent & Wiles, Table 3.) They do show, however, marked differences in these velocities for any particular structural condition depending upon the vertical angle of attack of the wind (Report of Messrs. Vincent & Wiles, Table 3 (Extended)).

This is an indication that the behavior of a long-span bridge, on which the angle of attack and the steadiness of the wind must vary constantly and considerably over the length of the bridge, cannot be judged by the behavior of a very short section model under any particular angle of attack.

Sufficiency of resistance to aerodynamic action must be based primarily on the experienced behavior of the structure under actual wind conditions. As experience and tests disclose, the resistance of long-span heavy suspension bridges to dynamic wind action is governed preponderantly by the weight of the structure suspended from the towers and by the stiffness, particularly the torsional stiffness, of the suspended structure.

It was due primarily to the addition of a second or lower lateral system on the Golden Gate Bridge, and the consequent large increase in torsional resistance, that the previous excessive motions under dynamic wind action were reduced to much smaller, so far unobjectionable, amplitudes.

Wind velocities of up to about 60 mi. per hr., with resulting unobjectionable motions, have been observed on the Golden Gate Bridge since the lower lateral system was added whereas, before, excessive motions were produced at much lower wind velocities.

As pointed out in the report of the Board of Engineers of January 2, 1953, there still exists the possibility, although remote, that under higher wind velocities more severe motions might be produced and that additional stiffening, such as is indicated in that report, might have to be installed.

This situation would not be appreciably altered by the addition of rapid transit. The somewhat adverse effect of rapid transit installation on the aerodynamic behavior, as indicated by the model tests, would probably be offset by the narrowing of the sidewalks and the increase in dead weight.

Earthquake Forces

We concur in Mr. Gronquist's conclusions that the Golden Gate Bridge is adequately stable and resistant against earthquake forces with or without rapid transit.

As in the case of live loads assumed for the design of a large bridge, the "permissible stresses" in the various members are necessarily assumed, partly on the basis of established practice, partly on individual judgment. They depend in the first place on the qualities of the material; for steel primarily on its yield point and ultimate strength determined by specimen tests or by tests of large size members.

A considerable margin of safety must be provided between the permissible stresses and the above strength qualities to cover many, more or less uncertain influences, such as increases in highway loadings, secondary stresses at connections and other points of flexure, uneven distribution of stress in large or compound sections of members, such as cables, anchorage chains, tower shafts, etc., impact influences, magnitude of variations in stress intensity under changing loads, frequency of these changes under traffic and effect on the fatigue of the material and the connections; also possible deterioration by corrosion in places where painting is difficult or impossible.

Experience on large bridges has demonstrated that such fatigue and other effects are more severe under rapid transit traffic than under highway traffic which is composed of a large number of widely distributed load units and produces smaller and less frequent vibrations and deformations of the structure.

Stresses in Wire Cables

The cables of the Golden Gate Bridge are composed of cold drawn wire of a specified minimum yield point of 160,000 lbs. per sq. in. and a minimum ultimate strength of 220,000 lbs. per sq. in. For this type of steel wire the yield point is a somewhat arbitrarily selected strength criterion. The ultimate strength is the more significant quality upon which the safety of the cable depends.

In the original design of the bridge a permissible maximum axial cable stress of 82,000 lbs. per sq. in. was adopted. This value of permissible stress for the same type of wire has been used in the design of the great majority of wire cable suspension bridges of medium and long span built in the last thirty years.

In the design of the Narrows Bridge in New York, now under construction, it was considered fully justified to raise the permissible stress to 86,000 lbs. per sq. in., partly because the large number of tests made previously proved the wire to be of uniformly excellent quality, exceeding the specified minimum strength in the average by about 6%, partly because that bridge will carry only vehicular traffic and, with a 12 lane capacity, has a suspended dead load of 36,800 lbs. per ft. of bridge which is over 88.5% of the total design load and probably well over 90% of the total load under all actual traffic conditions. Under such cir-

cumstances, if it should ever become necessary, the axial cable stress could, within reasonably conservative limits, be raised up to 90,000 lbs. per sq. in.

The Golden Gate Bridge has about the same span length and, as long as it carries only highway traffic has about the same dead load to total load ratio. A similarly increased axial unit stress to 90,000 lbs. per sq. in., if that should ever become necessary or desirable, could still be considered conservative.

The maximum axial cable stress in the bridge as it exists today, with the moderate reserve dead load and the highway design load we have assumed, combined with the temperature change, is 86,000 lbs. per sq. in., or still well within the above mentioned conservative limit for highway traffic.

If the Golden Gate Bridge should have to carry rapid transit traffic, however, a permissible stress of not more than 86,000 lbs. per sq. in. should be considered as an appropriate equivalent limit for the reasons previously set forth, namely greater and more frequent stress fluctuations and deformations of the structure.

Mr. Gronquist considered an axial stress of 90,000 lbs. per sq. in. permissible, while Mr. Paine restricted this limit also to 86,000 lbs. per sq. in.

The calculated axial cable stress under rapid transit operation is 97,750 lbs. per sq. in. or nearly 12,000 lbs. per sq. in. higher than what we consider the conservative limit for this stress under rapid transit operation. The minimum axial stress might be as low as 84,000 lbs. per sq. in. and the relatively frequent stress fluctuation, therefore, as high as 14,000 lbs. per sq. in.

With bending stresses in the cables at the saddles on top of the towers under the increased deflections of the cables produced by rapid transit, both the maximum stress and the stress fluctuation in the top wires would be considerably greater than the above mentioned values.

Because of this overstressing of the existing cables we consider the installation of rapid transit inadvisable. A strengthening of the cables to reduce the stresses is obviously impracticable.

Stresses in Anchorage Chains

The anchorage chains in the Golden Gate Bridge are composed of forged and heat treated eye-bar links. The critical strength properties of such steel are both the yield point and the ultimate strength determined on full-size bars.

The originally specified minimum values of these properties for any individual test bar were respectively 50,000 and 80,000 lbs. per sq. in. of the shank section. The minimum values attained in individual test bars were 50,400 and 79,700 lbs. per sq. in. respectively.

The originally specified permissible axial design stress was 27,000 lbs. per sq. in. Due to the practical impossibility of obtaining a uniform distribution of the anchorage stress among the various bars and to secondary stress effects from deformations a variation of stress in individual bars of 20% or more must be expected in the actual structure. Such variations were found by a series of stress measurements during the construction of the George Washington Bridge.

For highway traffic only, with the assumed reserve dead load, together with temperature effect, the calculated axial stress is 27,000 lbs. per sq. in. and therefore not over the originally specified permissible stress.

Under rapid transit operation the calculated stress under the assumed loads will reach 30,300 lbs. per sq. in. or 3,300 lbs. per sq. in. beyond what we consider the permissible limit for rapid transit operation. Because of the aforementioned nonuniform stress distribution the unit stress in individual bars might be as high as 36,000 lbs. per sq. in.

Mr. Gronquist calculates the maximum average stress to be 27,800 lbs. per sq. in. which he considers permissible. Mr. Paine considers the anchorage chains adequate as long as the cable stress does not exceed 86,000 lbs. per sq. in. (This corresponds to a maximum allowable of 27,000 lbs. per sq. in. in the chains).

We conclude that rapid transit installation would overstress the anchorage chains.

Stresses in Tower Shafts

The tower shafts of the Golden Gate Bridge are columns of exceptionally large sections, composed of a multitude of individual rolled plates and shapes. Together with transverse struts and bracing between the two shafts of a tower they form rigid frames laterally, but are sufficiently flexible in the planes of the cables to bend under the longitudinal motions of the cable saddles due to variations in load and temperature.

The towers are subjected to a complex combination of dead and live loads, temperature changes, transverse and longitudinal static wind forces, possible earthquake effects and dynamic wind action. These forces produce a constantly changing combination of axial compression and bending stresses for which different permissible limits are set up. The preponderant stress is compression, for which the yield point of the steel composing the shafts is the important safety criterion. An ample margin of stress below this point must be maintained. Although in the original design unusually refined stress calculations were made and care was exercised in the fabrication and erection of the towers to secure

perfect workmanship, considerable variations from the calculated stresses must be expected in the actual structure.

The material in the tower shafts is partly carbon steel, for which a yield point of 36,000 lbs. per sq. in. was specified, and partly silicon steel with a specified minimum yield point of 45,000 lbs. per sq. in. Specimen tests showed average yield points for all shapes of 40,800 lbs. per sq. in. for carbon steel and 52,400 lbs. per sq. in. for silicon steel.

For a combination of dead load, live load of unrestricted length and temperature change, the originally specified maximum permissible stresses in lbs. per sq. in. were as follows:

Axial stresses: 14,000 for carbon steel, 18,000 for silicon steel.

Axial and bending: 18,000 for carbon steel, 24,000 for silicon steel.

These permissible values should not be exceeded when rapid transit is added.

In portions of the tower of an aggregate height of about 400 ft., including the entire portion between 80 ft. and 300 ft. above the base, the calculated stresses are found to exceed these values. For the most critical sections the calculated stresses and the assumed permissible stresses are as follows:

At about 80 ft. below top of tower for combination of dead load, live load and temperature:

Kind of steel: Silicon

Kind of stress and permissible value: Axial, 18,000 lbs. per sq. in.

Calculated stress: Axial, 20,200 lbs. per sq. in.

Excess over permissible stress: 2,200 lbs. per sq. in.

At about 300 ft. below top of tower for the same combination of forces:

Kind of steel: Carbon

Kind of stress and permissible value: Axial and bending, 18,000 lbs. per sq. in.

Calculated stress: Axial and bending, 20,400 lbs. per sq. in.

Excess over permissible stress: 2,400 lbs. per sq. in.

Combinations of the above forces with static wind were not investigated but, with higher permissible stresses for such combinations, would probably not be critical for the tower shafts. Due to uneven distribution of the stresses within the very large composite shaft sections and to secondary stresses at

the strut connections, the actual stresses might at certain points be considerably higher than the above calculated values. Reinforcing of the tower shafts, to bring the stresses within permissible limits, would be impracticable.

As a result of his analysis, Mr. Gronquist concludes that "the towers will require no reinforcing for the addition of rapid transit." We agree substantially with Mr. Paine's conclusion that rapid transit would increase the stresses in the tower shafts by 5 to 10% over the allowable ones.

Stresses in Stiffening Truss Chords

The most critical stresses in the chords of the stiffening trusses are caused by a combination of the dead load added to the original design dead load, a live load of restricted length, temperature and lateral wind force.

The material of the chords is silicon steel for which a minimum yield point of 45,000 lbs. per sq. in. was specified. Specimen tests showed an average value of 53,300 lbs. per sq. in.

Maximum chord stresses are produced by combinations of either dead plus live load plus temperature plus one half wind or dead load plus one half live load, plus temperature plus wind. For either of these combinations permissible unit stresses of 1.33 times the normal stresses are specified, namely 25,900 lbs. per sq. in. for compression on gross section and 32,000 lbs. per sq. in. for tension on net section.

Under rapid transit these permissible stresses are exceeded by the calculated stresses in about 40% of all chord members in the center span. The maximum calculated compression stresses are 29,300 lbs. per sq. in. or 3,400 lbs. per sq. in. in excess of the permissible value. The maximum tension stresses are 36,800 lbs. per sq. in. or 4,800 lbs. per sq. in. higher than the allowable maximum. The chord stresses in the side spans are within permissible values.

Due to the considerably increased vertical deformations of the stiffening trusses under the added loads there will also be superimposed on the above primary or axial stresses appreciable secondary or bending stresses at the connections between chords and web members.

It is evident, therefore, that if rapid transit were installed a large percentage of the chord members of the stiffening trusses would have to be strengthened.

Both Mr. Gronquist and Mr. Paine report that strengthening of the chords would be necessary to various extents.

While the strengthening of the stiffening trusses appears possible a much more elaborate study of the entire deck structure would have to be made to

determine the extent of the necessary reinforcement. It is evident, however, that this operation under traffic would be extremely difficult, time consuming and with the greatest care it would be practically impossible to secure any degree of accuracy in the resulting stress distribution.

The range in stress from maximum to minimum value, which would occur in the chords of the stiffening truss under each passage of a train is large enough to be of concern if the present trusses were not materially strengthened. The fatigue life for silicon steel is no greater than that of carbon steel. Hence stresses applied a great many times can be more deteriorating to silicon steel members if stressed to a higher level than would be used for ordinary carbon steel members.

Under highway loading a change in stress from near minimum value to near maximum value occurs much less frequently than under rapid transit loading. Hence more conservative stresses must be used to provide against possible fatigue failures in the stiffening truss members under rapid transit loading. The added material required for this purpose can be considerably greater than the estimates of added dead load that have been made by Mr. Gronquist.

Stresses in Suspenders

Each suspender consists of four single-part wire ropes of 2 11/16 in. diameter. Each of the two two-part ropes is looped around the cable band at the top and its lower ends are connected to sockets attached to the deck structure.

The specified minimum ultimate strength was 1,100,000 lbs. for a double-part rope and 606,000 lbs. for a single-part. The average tested strengths were 1,327,000 and 703,000 lbs. respectively.

It is customary to allow a total stress in the rope of one third of the double-part strength or one quarter of the single-part strength. In this case the permissible total rope stresses would be 442,000 and 176,000 lbs. respectively.

With rapid transit the calculated stresses from a combination of dead load, live load and impact are 375,000 and 187,500 lbs. respectively.

The permissible single-part stress, therefore, is somewhat exceeded. However, since there is a tendency to equalize the stresses in the four parts of a suspender under load and there is a considerable reserve strength in the more significant double-part rope, the suspenders may be considered adequate under rapid transit.

Both Mr. Gronquist and Mr. Paine reported the same conclusion.

Stress Condition in other Parts of Suspension Spans

We have made no calculations to determine the stress condition in secondary members, such as stiffening truss diagonals, top and bottom laterals, tower bracing, etc., but it is likely that many of these would require reinforcement or replacement if rapid transit were permitted on the bridge.

The tower piers, anchorage blocks and their foundations have undoubtedly sufficient strength and stability to safely sustain the additional forces which would be imposed by rapid transit.

DEFLECTIONS AND THEIR EFFECT ON NAVIGATION CLEARANCES AND ON PROFILE OF BRIDGE ROADWAY

We have calculated the deflections of the cables in the center span which would be caused by the additional dead load and the live loads from highway and rapid transit, as assumed herein.

Under a combination of these loads and a temperature of 110° F. the deflection at the center of the center span from the original dead load position would be 19.3 ft. Mr. Gronquist and Mr. Paine reported 18.2 ft. This would cause an encroachment of up to about 9 ft. on the 220 ft. under-clearance originally required by the War Department.

A ruling with respect to such encroachment would have to be obtained from the present Department of the Army. If it should not be permitted installation of rapid transit would not be possible.

The added deflections of the deck structure suspended from the cables in both center and side spans, combined with increased elongations of the suspender ropes, especially the long ones near the towers, would cause appreciable and objectionable changes in the roadway profile with sharp kinks at the towers. They would also cause excessive stresses in the stiffening truss diagonals near those points.

To offset these effects, at least in part, Mr. Gronquist proposes to jack up the stiffening trusses and insert shims between the suspender sockets and their bearings. This or any other method would undoubtedly be feasible, but would involve very tedious, delicate and costly operations. They would have to be made in a series of gradual steps and it is doubtful that satisfactory adjustments could be attained, especially because they would have to be made under traffic.

PRACTICABILITY OF STRUCTURAL CHANGES, ADDITIONS AND ADJUSTMENTS

We have not studied in detail the many structural changes, additions and adjustments which would have to be made in the existing structure, nor the

methods which would be necessary to secure satisfactory results. They would require much more detailed studies than have been made so far to establish their practicability. Certain of these changes, such as the expansion dams of the highway deck, could not be accomplished without serious interference with the traffic. Others would have to be limited to night hours when traffic is light and temperature conditions are steady. From among the many such changes the following are mentioned as requiring most careful detailed study:

Mr. Gronquist proposes to reinforce the central part of the transverse struts which are to carry the rapid transit track stringers by replacing the latticing by solid web plates. This would create a very narrow, deep tube and, although a few small handholes are provided, would not permit adequate inspection and maintenance. Its redesign might also involve changes in the laterals and, since the many intersections of the laterals and the new track stringers would require extensive cutting and resplicing of parts of the laterals, it might be found that a complete dismantling of the present lateral system and its replacement by a properly designed lower deck structure would not only be more practicable, but also more economical.

No calculations have been made to determine the effect of torsional forces caused by one-sided train and highway loads on the deck structure. Such investigation might disclose the necessity for further reinforcement in the members of the upper as well as the lower lateral system and its connections.

Due to grade changes at the existing expansion dams in the highway deck, and to greater horizontal motions at these dams caused by the greater vertical deflections, as well as by braking and traction forces caused by trains, the expansion dams would very likely require complete redesign and replacement.

The feasibility of increasing the sections of the stiffening truss members would also require more careful study and might divulge that much more material than has been estimated would have to be added in order to secure effective strengthening.

The possible loosening of the cable wire wrapping, the retightening of the cable bands as a result of the increased stretching of the cable wire and the effect of greater angular changes of the cables at the tower saddles would require careful study.

All these detailed design studies would likely divulge further necessary increases in the dead weight of the superstructure and, in turn, increase the already excessive calculated stresses in the major carrying members under rapid transit operation.

APPENDIX—COMMENTS ON POSSIBLE FATIGUE EFFECTS IN GOLDEN GATE BRIDGE

by

N. M. Newmark and W. H. Munse

INTRODUCTION

Repeated loadings from traffic or wind cause fluctuation in the stresses in the various parts of the bridge. When the stresses vary over wide enough ranges, these variations in stress may cause cracks in members and eventual fatigue failures. The parts of the structure where fatigue might be a problem include the bridge cable wires, the anchorage eye-bars, and the connections in the stiffening trusses. Comments are made herein on the possibility of fatigue failures developing in these parts of the bridge.

Test data are also reported here on the fatigue strength of sections of cable wire, both from the bridge and from wire not previously used in a suspension bridge cable.

The conclusion is reached that the Golden Gate bridge is adequately safe from fatigue failures under present highway loadings. It is further concluded that, under the contemplated rapid transit loading, fatigue will not be a problem in the cables or in the eye-bar anchorages. However, there is a possibility that fatigue failures may develop after a period of years in the connections in the stiffening trusses unless the fluctuations in stress in the stiffening truss members are held to a minimum. If material is added to the present stiffening truss, it must be added in such a way that the additional material participates fully in carrying the load in order to avoid fatigue failures.

Since fatigue failures are likely to occur only after some twenty to fifty years of additional service under rapid transit traffic, provision for future inspection at periodic intervals must be made to assure safety against deterioration and damage in the bridge.

Because corrosion accelerates and augments the effects of repeated stress or fluctuations in stress, where fatigue conditions are important particular attention must be paid to avoiding the influence of corrosion, even that arising from continual or repeated infiltration by rain water.

POSSIBLE FATIGUE IN CABLE WIRE

Fatigue tests were conducted on bridge cable wire by Shelton and Swanger[1] at stresses that encompass the usual operating conditions for suspension bridge cables. In their tests it was found that the range of stress at the fatigue limit was nearly independent of the mean stress or the minimum stress for stresses ranging all the way from a minimum of 25,000 psi to a maximum of 175,000 psi. Over this complete range it was found that the fluctuation in stress at the fatigue endurance limit was approximately 50,000 psi for wire with an ultimate strength of approximately 220,000 psi.

The data presented by Shelton and Swanger also give results for shorter fatigue life conditions. For example, a fluctuation in stress of approximately 70,000 psi was attained for approximately 100,000 cycles of repeated stress, as compared with a fluctuation of 50,000 psi for over 2,000,000 cycles.

Limited test data on the effects of corrosion[2] indicate a range of stress of the order of about one-third that without corrosion, for wire subjected to a continuous water spray. This is consistent with other data on corrosion fatigue in steel members and elements.

Because of the possibility that a bending condition may exist at the cable strand shoes, it was decided to perform additional tests on bridge cable wire reproducing the effects of stressing the wire over a two-foot diameter sheave. Tests were made on two specimens of wire cut from the cable anchorages on the Golden Gate bridge, and on one specimen of virgin wire of the same general metallurgical constitution. The tests were made in the University of Illinois fatigue-testing machine, which was modified to permit fatigue testing of 96-in. lengths of wire supported at the middle by a 24-in. diameter sheave, and anchored at the ends by special wire gripping devices. This fixture was intended to reproduce approximately the conditions existing at the strand shoes and the cable anchorage.

Although the wire was initially strained in bending, because of the difference in curvature of the free wire and the wire under stress around the sheave, these conditions approximately reproduce those in the actual bridge cable. At a stress of the order of about 10,000 psi the load-strain relationship in the wire became nearly linear and further bending of the wire did not occur. Consequently, in the testing machine, and probably in the bridge, additional bending stresses in the cable anchorages are small. However, additional bending stresses in the bridge in the cable wire as it passes over the saddles at the tower may not be so small.

All of the fatigue tests were made with a minimum stress of 84,000 psi, which is estimated to be the minimum stress in the cable with the added dead load placed on the bridge. Specimen No. 1, of virgin wire, was subjected to 100,000 cycles of loading at a stress range from 84,000 to 110,000 psi, and then to 221,800 cycles of a stress range fluctuating from 84,000 to 144,000 psi, until failure occurred about 1 in. away from the grips. This result is consistent with previous test data.

Specimen No. 2, consisting of a section of wire from the bridge cable, was subjected to loading with a stress ranging from 84,000 psi to 114,000 psi for 2,000,000 cycles, then to a stress ranging from 84,000 to 144,000 psi until failure occurred at 210,400 cycles of additional loading. Failure occurred at about 4 in. away from the grips. This is also considered to be in good agreement with previous tests and with test data on Specimen No. 1, because of the fact that previous loading history with relatively small ranges of stress appears to have little if any harmful influence on the fatigue resistance of specimens subjected to subsequent higher stress.

Specimen No. 3, also a wire from the Golden Gate bridge, was subjected to a stress cycle from 84,000 to 124,000 psi for 2,000,000 cycles, and then to a stress ranging from 84,000 to 144,000 psi for 2,500,000 cycles of additional loading without failure. This increase in the additional number of cycles indicates the so-called "training" or "coaxing" effect of a moderately high stress range, just below the endurance limit, on subsequent stressing at a level only moderately higher.

The data from these tests, and that reported in the literature for similar wires, are in excellent agreement and appear to confirm the conclusion that the previous loading history had little or no effect on the fatigue resistance of the bridge cable wire.

POSSIBLE FATIGUE EFFECTS IN EYE-BARS

Fatigue data on eye-bars are not directly available. However, data are available from tests unreported in the literature on simple pin-connected links, and on riveted joints with relatively loosely fitting rivets. From these data for a variety of steels, it is estimated that the fluctuation of stress at the endurance limit, corresponding to 2,000,000 cycles or more of repeated stress, is about 10,000 to 12,000 psi, with the smaller values being applicable to higher mean or minimum stresses. This range in stress is that in the head rather than in the body of the eye-bar. This stress fluctuation capability is about three to four times as great as the estimated maximum range in stress for the eye-bars in the bridge; hence there should be no problem of fatigue in these members.

POSSIBLE FATIGUE EFFECTS IN STIFFENING TRUSS CONNECTIONS

A great deal of data is available on fatigue in riveted connections.[3,4,5,6,7] These data are available for a variety of details, a number of different loading conditions, and several different types of steel. However, since the size and details of the connections and the loading conditions in the bridge differ markedly from those used in the laboratory tests, care is necessary in evaluating and applying the data to an actual structure.

From a study of the various data, the following conclusions are reached regarding the fatigue lives of connections of a type similar to those in the bridge.

(1) At 2,000,000 or more cycles of load, no fatigue advantage is realized with higher strength steels.

(2) The fatigue resistance at 2,000,000 or more cycles of application of load corresponds to a stress fluctuation, in either complete reversal or in the range from zero-to-maximum tensile stress, of approximately 20,000 psi.

(3) At 100,000 cycles, there is a slight advantage for low alloy or silicon steels over ordinary carbon steel, but the difference is not great.

(4) The fatigue resistance at a life of 100,000 cycles for connections of the type considered, and for silicon steel, is estimated to be, both for complete reversal or for a range from zero-to-tension, a total fluctuation of approximately 30,000 to 35,000 psi.

For conditions of stress fluctuation in which large fluctuations are combined with small fluctuations, there is a cumulative effect. Nevertheless, fluctuations corresponding to less than a range of about 15,000 psi for any cycle would have no influence on the endurance.

Because the loadings that combine to give maximum stresses in the stiffening truss are quite complex, it is difficult to estimate the number of fluctuations for the various ranges of stress that might occur. Considering wind gusts of moderate intensity, and normal highway and rapid transit loading, a very rough estimate has been made indicating that stress fluctuations of the order of more than 40,000 psi would occur rarely, possibly not even as often as several times per year. However, stress fluctuations of the order of 25,000 to 30,000 psi might occur several times per month, and fluctuations of the order of 20,000 to 25,000 psi might occur on the average as often as about ten times per day. Stress fluctuations of the order of 10,000 to 20,000 psi might occur as many as 100 to 200 times per day. These are estimates made for rapid transit loading

as well as highway loading on the bridge. However, if only highway loading is permitted on the bridge, the maximum stress fluctuations would occur much less frequently, and their range would be reduced.

It is concluded from these estimates that under highway loadings of the present type there is no reason to expect fatigue failures in the stiffening trusses. With rapid transit loading added, however, there may be difficulties with fatigue occurring in the present stiffening trusses after a number of years of service. This possibility can be reduced only by reducing the magnitude of the stress fluctuations as a result of most careful addition of material in the chords and web members.

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